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Performance of a 30 m Deep Instrumented Diaphragm Wall

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SYNOPSIS This paper summarises the performance of the second instrumented research section of the Rio de Janeiro underground. The first aim of this programme was to establish a simple procedure to design diaphragm walls in soft clay soils. The model of a beam on elastic supports was chosen due to its simplicity and usefulness in the prediction of the wall behaviour. The concrete wall was built using the slurry trench technique and was instrumented to measure the concrete strain, the load on the reinforcement bars, the horizontal displacement and the horizontal earth pressure. It was measured the loads in the two level of struts. The geotechnical profile consists of a soft layer between two layers of sand. The surrounding soil received instruments to measure the pore water pressure, the horizontal and vertical displacements at different locations. This paper presents the comparison between predicted and measured values during the second stage of excavation.

INTRODUCTION

The Civil Engineering Department of the Coordination of the Post-graduation Programmes in Engineering (COPPE) of the Federal University of Rio de Janeiro (UFRJ) instrumented and observed the behaviour of the Rio de Janeiro underground construction in nine places. The instrumentation was undertaken in the majority of the places to reduce risks of damage to the surrounding buildings. Two test sections were instrumented to furnish data to establish a simple procedure for designing diaphragm walls in clay soils, as well as to improve other methods of design and develop instrumentation techniques. This paper summarizes some of the field measurements made at the second instrumented section and compares the predicted and measured performance during the second stage of the excavation works. More details about the performance of this test section can be found in the DSc thesis of the author (Soares, 1981).

SOIL CONDITIONS

Knowledge of the soil properties was obtained in two distinct programmes. The first investigation programme was carried out to design the wall and the second to give additional information about the soil parameters thus permitting more realistic analysis of the behaviour of the instrumented test section. The first programme consisted of borings with standard penetration tests every 0.5 m and laboratory index tests on disturbed samples. Conventional compressibility and shear strength laboratory tests were carried out on 50 and 100 mm shelby samples. Figure 1 shows some results from this programme. The most important layer for the design is the clay layer 15 m deep with SPT about 2 and cone point resistance 60 kN/m^2. On that occasion the Designer decided to adopt for safety and undrained shear strength equal to 30 kN/m^2.

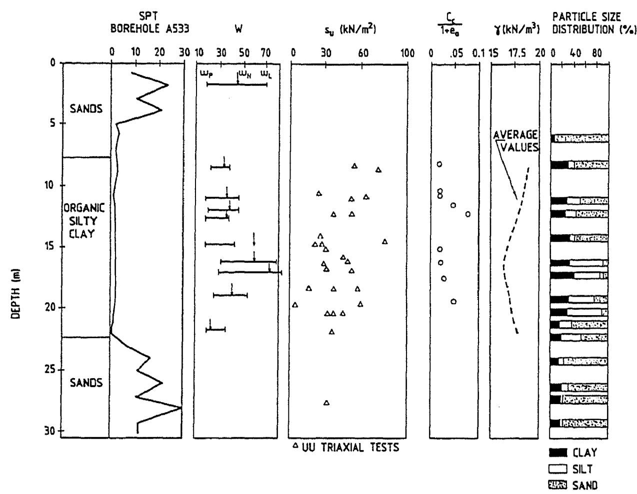

Fig. 1. SOIL PROPERTIES SUMMARY FIRST PROGRAM OF INVESTIGATION

The 120 cm thick, 30 m deep concrete wall supported at two strut levels was designed by applying the standard design procedures used for by the Rio de Janeiro Underground Company. These procedures were based on the free earth support and fixed earth support methods. At the same time it was decided to instrument a typical section to give more data to establish an improved procedure applicable to the design of concrete walls in soft clay soils.

The second programme of soil investigation with two phases was started at the same time as the installation of the instruments and excavation in the area. The first phase of this programme consisted of:

- boreholes for installation of the instruments and soil sampling with some standard penetration tests

- conventional laboratory tests on undisturbed soil samples (shelby and block)
- standard laboratory tests on concrete samples obtained from the wall
- in situ tests - deepsoundering, vane and pressuremeter

Figure 2 shows data obtained from this first phase of the second programme. The distribution of the undrained shear strength adopted in calculations in this paper is given in Table 1. The values of the SPT's in the different borings and at different depths were valuable in establishing this adopted undrained shear strength distribution. It was impossible to improve the quality of the site investigation because the work was performed by a Private Contractor not under our control. The adopted values presented on Table 1 are believed to represent the most likely undrained shear strength profile.

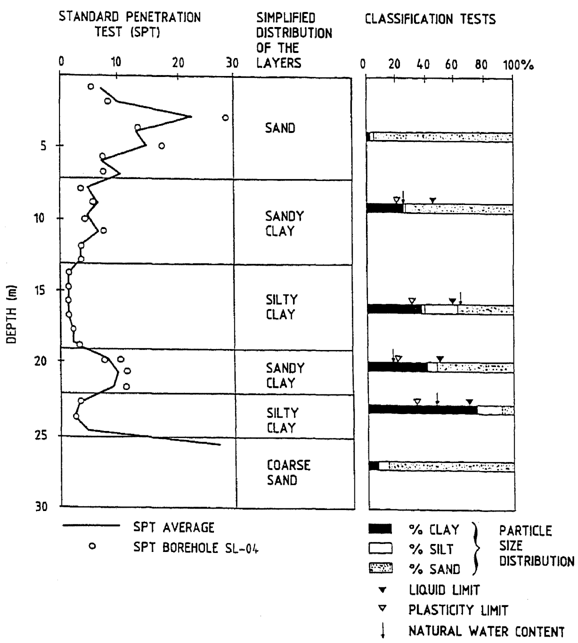

Fig. 2. DEFINITION OF THE SIMPLIFIED DISTRIBUTION OF THE LAYERS

TABLE I.

Depth (m)	Layer	Average undrained shear strength (kN/m²)		Su (kN/m² Adopted
		Routine contractors UU Triaxial Tests	High quality block samples UU Triaxial Tests	
7-13	Sandy Clay	45	75	80
13-19	Silty Clay	20	-	40
19-22	Sandy Clay	45	-	90
22-25	Silty Clay	40	-	80

The second phase consisted of special laborato: tests on undisturbed samples to establish the stress-strain behaviour of the soils on differe stress paths. These tests are being carried o at our laboratory COPPE-UFRJ. Initial results showed that the adopted undrained shear streng represents satisfactorily this soil property. The relation between the initial modulus of deformability in undrained extension tests and the undrained shear strength does not vary muc with OCR and for practical purposes a value between 200 and 300 can be utilised for this relationship. More information about the soil parameters can be found on Lins (1980) and Correa (1981).

FIELD INSTRUMENTATION

The instrumented area was chosen in a place sufficiently distant from buildings to make th analyses of the data easier.

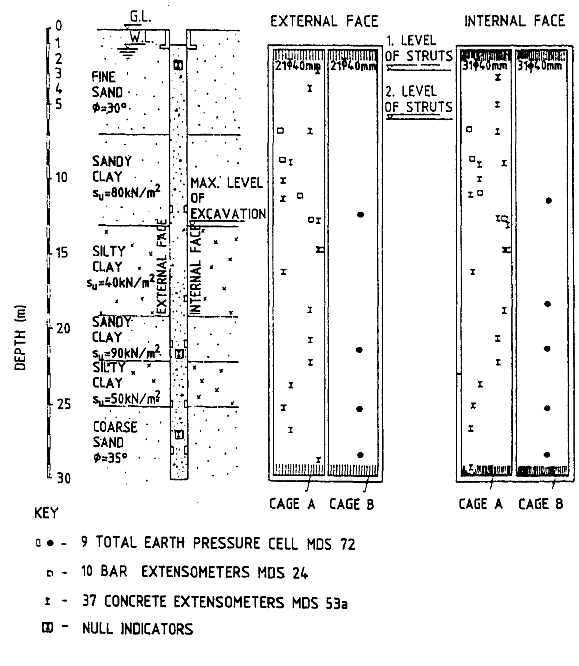

Fig. 3. PANEL INSTRUMENTATION

ure 3 shows the instrumented diaphragm wall el with the transducers to measure the crete strain at 16 sections, reinforcement loads at 5 sections and total horizontal th pressure at 9 positions. The installation cedures for these instruments has been cribed elsewhere (Soares, 1983).

ure 4 shows the soil instrumentation centrated near the instrumented panel. Pore ssures were measured near the wall inside and side the excavation by pneumatic piezometers.

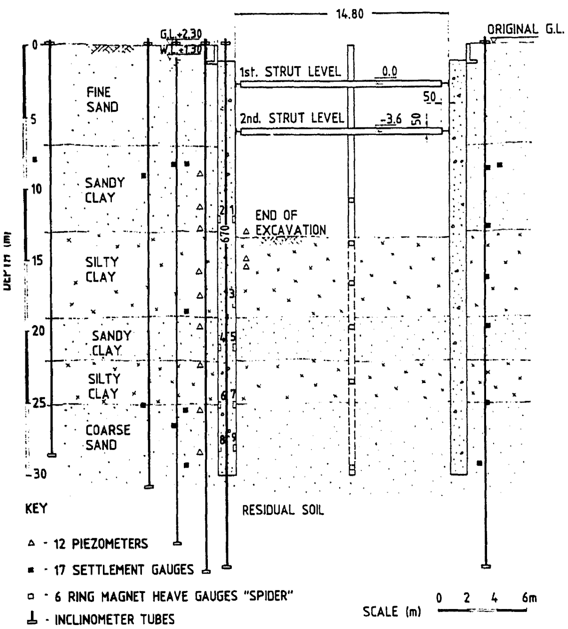

Fig. 4. SOIL INSTRUMENTATION

ıe vibrating wire piezometer was installed ıside the excavation near its final level. ·ven inclinometer tubes were installed, six in ıe soil behind the wall at different distances ›art and one in the centre of the wall. The ›rizontal movements of the tubes were measured .th digitilt inclinometers models 50301 and)306 manufactured by SINCO. The settlements at .fferent positions were measured by means of ırvey control and the bottom heave on the ›ntre of the excavation at different depths was ›asured with ring magnet gauges. Strut loads ›re monitored by four vibrating wire ‹tensometers distributed in two sections.

CONSTRUCTION AND MEASUREMENTS DURING THE SECOND STAGE OF EXCAVATION

The 1.2 m thick and 30.0 m deep concrete diaphragm wall was built by the slurry trench technique excavated in panels 7.5 m long. The bentonite slurry used has a specific gravity about 10.5 kN/m^3. Each panel had a preassembled steel reinforcing cage weighing 350 kN.

The excavation between the concrete walls was done in three stages; excavation started on December 8th, and took 85 days to reach the final level 13 m deep. Figure 5 portrays the progress of the construction. The walls were supported at two levels by steel beams 1.80 m apart and not preloaded.

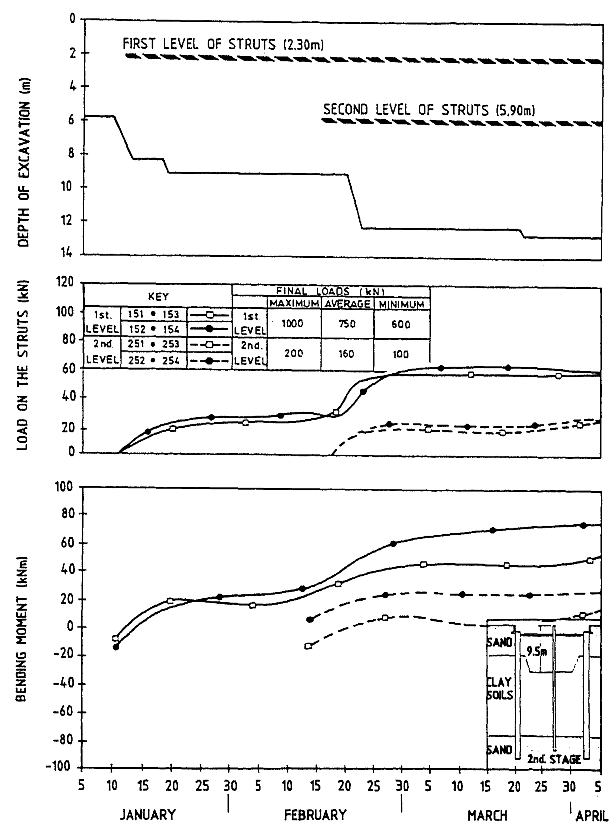

Fig. 5. LOADS AND BENDING MOMENTS MEASURED ON THE STRUT 5.

Figure 5 also shows the loads and the bending moments measured in one of the seven instrumented struts. There was very good agreement between the measured loads at the two ends of the struts. The maximum average and minimum loads at the end of excavation are 560, 420, 340 and 110, 90, 60 kN/m for the first and second bracing level respectively.

Figure 6 shows the concrete strain measured at the end of the second stage of excavation (before installation of the second level of struts). From these measurements, the bending moments in each section of the wall were calculated assuming a linear distribution of the strain at each section and a value for the Young's modulus of the concrete of 30×10^6 kN/m^2. This value of elasticity modulus was slightly higher than the average (28×10^6 kN/m^2) obtained in the static tests on concrete samples obtained from the wall. The value of this modulus from dynamic tests showed a good homogeneity of the concrete with the depth of the wall and the average value was 36×10^6 kN/m^2.

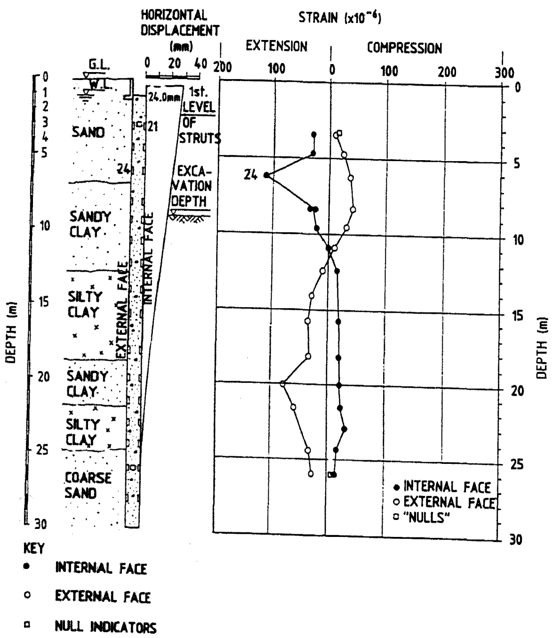

Fig. 6. CONCRETE STRAIN DURING THE SECOND STAGE OF EXCAVATION.

COMPARISON BETWEEN PREDICTED AND MEASURED VALUES

The importance of establishing a simple design procedure was the aim at the first stage of the analysis of the instrumentation data. The simple procedure must be able to predict the loads on the struts, the distribution of the bending moment and the horizontal displacement of the wall. The settlement of the surrounding ground, could be estimated from the horizontal movements using an existing method (for example Caspe, 1966; Juca, 1981). This simple procedure could then be incorporated into the design approach suggested by Rio de Janeiro Underground System (RJUS).

Empirical and semi empirical methods such as t apparent earth pressure, free earth support an fixed earth support, were used to predict the behaviour of the instrumented section. The apparent earth pressure method as described by Terzaghi and Peck (1967) only estimates the maximum loads on the struts. The generalisati of this method as suggested by some authors do not give a reasonable prediction of the bendin moments in the wall. The use of the free and fixed-earth-support methods does not give good results either. It was suggested (Soares, 198 that these methods should not be applied to th design of diaphragm walls in clay soils.

The finite element method was not adopted, at this stage of the analysis, due to the difficulties of simulating excavation and in most field cases the necessary data is not available. The author believes that sophisticated analytical techniques may be no more appropriate than simpler solutions when these are used with engineering judgement.

From its first use it was seen that the model a beam on elastic supports (Winkler hypothesis could be useful in the prediction of the wall behaviour. Many suggestions were obtained frc Fages et al (1971a, 1971b and 1973), Dalerci a Torrigiani (1976) and Miyoshy (1977) and all available methods were used to estimate the horizontal coefficient of subgrade reaction.

The structural pattern of the wall was a beam supported by the struts and by the soil below the excavation level. These supports behave like elastic springs. The constants of the strut springs were determined from the measure loads and horizontal displacements. It is important to remember that the effective rigidity of the struts depends on the construction conditions. For preloaded struts this rigidity can be a tenth or less of the structural rigidity. For struts that have not been preloaded the effective rigidity can be much smaller even than this.

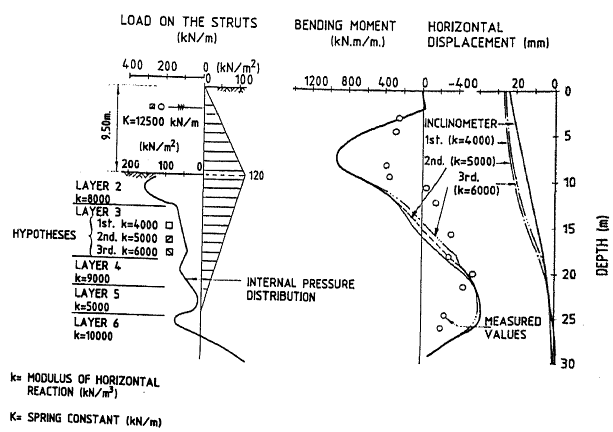

Fig. 7. COMPARISON BETWEEN MEASURED VALUES WITH THOSE PREDICTED BY MIYOSHY METHOD.

The suggestion of Miyoshy (1977) when applied to the Botafogo wall conditions gives the values shown on Figure 7. These calculated values overpredict the measured bending moments, the horizontal displacements and strut loads. Miyoshy (1977) considers the loading on the external face to be the active pressure up to the level of the excavation and from this point up to a length equal the width of the excavation a linear decrease of the loading (Figure 7). It was decided to reduce the load suggested by Miyoshy (1977), after verifying that the soil modulus did not have a substantial influence on the results (Figure 7).

Figure 8 shows the predicted values for a loading equal to the active pressure up to the level of the excavation. This condition gives a reasonable prediction of all measured values. These results show that there was a substantial restriction to the change of the stresses in the soil inside the excavation.

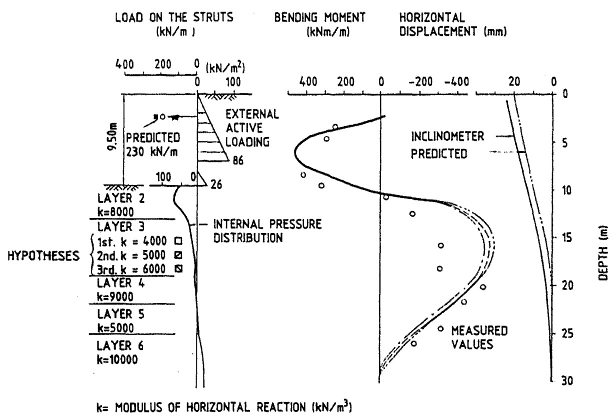

Fig. 8. COMPARISON BETWEEN MEASURED VALUES WITH THOSE PREDICTED WITH ACTIVE SOIL PRESSURE ABOVE THE EXCAVATION LEVEL.

Figure 9 shows the influence of the magnitude of the effective rigidity of the struts. The increase of this value increases the maximum bending moment above the level of the excavation and decreases the moments in the embedded length of the wall.

Figure 10 portrays the influence of the soil's modulus. The best value of this modulus was obtained when using the following relationship:

$$k\ (kN/m^3) = 100 \text{ to } 200\ Su\ (kN/m^2)$$

It was concluded that this value of soil's modulus corresponds to the undrained modulus of deformability from extension tests. Considering a safe relation between SPT and Su this horizontal soil modulus can be estimated by:

$$k = 1000 \text{ to } 1500\ SPT\ (kN/m^3)$$

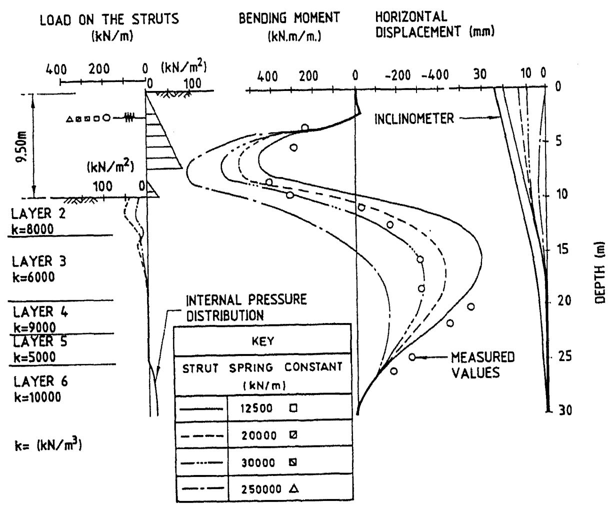

Fig. 9. INFLUENCE OF THE EFFECTIVE RIGIDITY OF THE STRUTS ON THE VALUES PREDICTED WITH ACTIVE SOIL PRESSURE ABOVE THE EXCAVATION LEVELS.

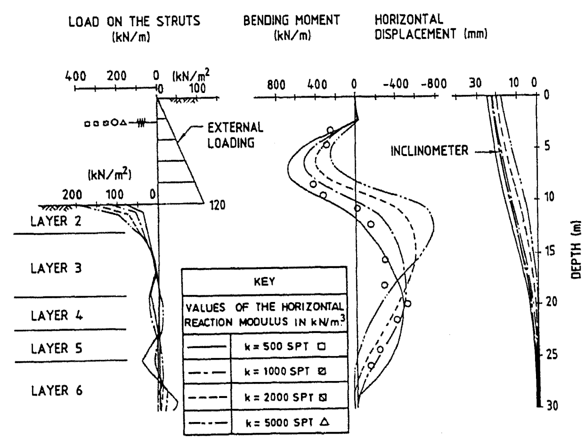

Fig. 10. INFLUENCE OF THE SOIL MODULUS ON THE PREDICTED VALUES

CONCLUSIONS

The measurement of bending moment, strut loads and horizontal earth pressures at the second instrumented section at the Rio de Janeiro Underground made it possible to develop a simple design procedure for diaphragm walls in a stratified soil profile with soft clay soils. The model of a beam on elastic supports has been used and the horizontal subgrade reaction modulus has been related to standard laboratory and field data.

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